GUIDANCE FOR IN-SITU SUBAQUEOUS CAPPING OF CONTAMINATED SEDIMENTS:

Appendix C: Case Studies on Geotechnical Aspects of In-Situ Sand Capping

by

Hoe Peter Ling
Dov Leshchinsky
Department of Civil Engineering
University of Delaware
Newark, DE 19716

Prepared for

U.S. Environmental Protection Agency
Great Lakes National Program Office
Assessment and Remediation of
Contaminated Sediment Program
Chicago, Illinois 60604

Monitored by

U.S. Army Engineer Division, North Central
Chicago, Illinois 60605-1592

Appendix C: Case Studies on Geotechnical Aspects of In-Situ Sand Capping

Introduction

Industrial activities have resulted in significant deposits of contaminated sediments in some US harbors and waterways. Remediation of these contaminated sites can be costly and technically difficult. In-situ sand capping has been identified as a feasible and cost-effective technique for on-site remediation. The extremely low shear strength of these sediments presents unique engineering problems. The geotechnical aspects of successful in-situ sand capping projects conducted in the U.S., Japan, and Norway are reviewed and compiled in this report. Geotechnical assessment of in-situ capping technique, based on bearing capacity and slope stability analyses, is made with reference to these projects. Usage of geosynthetic of adequate strength and hydraulic conductivity is recommended to improve the sand cap stability in case where extremely soft sediments are encountered. Recommendations leading to improvement of sand capping design are included.

Significant deposits of contaminated submarine sediments are found within the U.S., in and around the Great Lakes, typically as a consequence of industrial manufacturing activities. These materials are physically characterized by a low shear strength and high compressibility. They are easily transferred to the water column as a result of disturbance by natural currents and maritime activities. For example, propeller wash from the traffic and movement of powerful vessels at shallow depths are found to be a source of significant disturbance. The Army Corps of Engineers has been involved in developing technical guidelines related to remediation by dredging and capping (Palermo et al., 1993).

The level-bottom capping and contained aquatic disposal are two of the most common methods of isolating dredged contaminants in the U.S. Clean materials, such as sand, have been used to cap the contaminated sediments. The Army Corps of Engineers, New England Division, initiated the first sand capping project on dredged sediments in Central Long Island Sound (CLIS) Disposal Site at Connecticut in 1979, as part of Disposal Area Monitoring System (DAMOS). This is referred to as Stamford/New Haven Project in which contaminated sediments were dredged from Stamford and New Haven Harbor. The Stamford sediments were deposited as two mounds, one capped with sand (2.1-3 m thick) and the other with silt (3.9 m thick). A successful sand capping project was also reported for the Mud Dump Site in New York Bright in 1980 (O'Connor and O'Connor, 1983). The contaminated dredge material was capped with fine sediments from the Bronx River and Westchester Creek, then followed by sand from the Ambrose Channel. The cap was 1 m thick.

A comprehensive monitoring program was conducted when the Black Rock and New Haven Habors were dredged in 1983. Black Rock Harbor sediments were reported to be composed of organic silt and clay that were highly contaminated with oil, grease, heavy metals and PCB's. The dredged sediments from this site were placed in two mounds in CLIS Disposal Site and capped with silt from New Haven Harbor and sand from the nearby channel, respectively. A Field Verification Program (FVP) was also conducted on the uncapped sediments at the northeastern corner of CLIS site in order to evaluate the effectiveness of capping. A monitoring program was established for these sites and documented by SAIC (1984). Additional cases of sand capping projects may be found in Palermo et al. (1993).

Remediation of contaminated sediments by first dredging followed by disposal and capping at a site different from the source may not be the most economical solution. As the volume of contaminated material increases, an appropriate disposal site becomes limited. Risk of resuspension of contaminants into the water column increases by disturbance during dredging and disposal. Due to the extremely low shear strength of sediments immediately after dredging, cap placement is technically very difficult. This has led to use of different technology in Japan in which sand caps are placed directly over contaminated sediments without involving dredging (hereafter known as **in-situ capping**). The purpose of this report is to document the geotechnical aspects of several in-situ capping projects conducted in Japan, U.S. and other countries. The report also highlights cases in which geotextiles were used to improve stability of the sand caps placed on extremely soft sediments. Geotechnical evaluation of sand cap and foundation stability are made with reference to these case histories.

In-Situ Capping: Case Histories

Successful Japanese sand capping projects were conducted primarily on fishery grounds near the Seto Inland Sea (Figure C-1). This area has poor current circulation and is affected by heavy industrial discharges carried by several major rivers. **The red and blue tides have seriously affected the fisheries**. An experimental in-situ sand capping project was conducted in 1979 by the Port Construction Bureau of the Ministry of Transport. Since then, several other projects were conducted (see Table 1). Figure C-1 gives the individual location of these sites. Earlier studies related to in-situ sand capping projects tend to focus on the chemical and biochemical aspects of the sand-sediment-water column environment. It has, however, been recognized that success of this technique depends also on geotechnical considerations. The following is a description of a few of the well-documented cases with insight on geotechnical properties. Later, these cases are utilized for geotechnical evaluation.

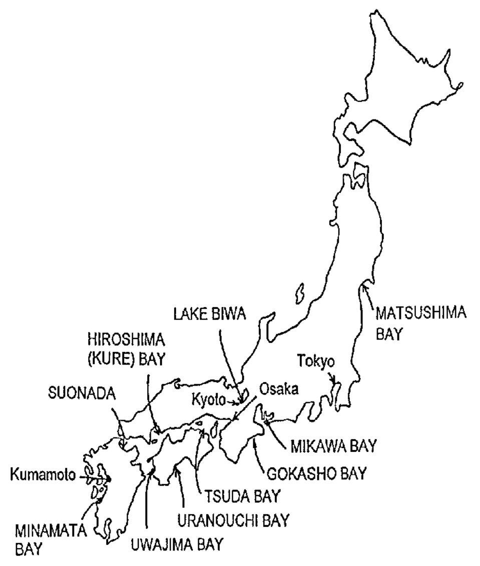

Figure C-1. Sand capping sites in Japan.

Table 1. Detail of Sand Capping Sites

SITE	YEAR	DEPTH BELOW SEA LEVEL (m)	CAP THICKNESS (cm)	AREA (10^3 m^2)	MAIN REFERENCE
Hiroshima Bay	1979	21	50	19.2	Horie
(Japan)	1980	21	30	44.8	(1991)
	1979		15	1.2	P.C.B.
Uranouchi Bay		13-14	15	10.0	(1994)
(Japan)		6-9	20	7.4	
Suonada Bay					P.C.B.
(Japan)	1986	1	30	0.9	(1994)
	1987	5	50	15.0	P.C.B.
Mikawa Bay			40-100	9.6	(1994)
(Japan)			40-100	4.5	
Minamata Bay[1]	1988	2-10	80	324.7	Namba
(Japan)					(1994)
	1991	10-15	50	212.8	P.C.B.
Tsuda bay	1992	10-15	50	114.0	(1994)
(Japan)	1993	10-15	50	91.2	
Lake Biwa[2]					Gomyoh et al.
(Japan)	1992	1.5	20	24.2	
Matsushima Bay[2]					(1994)
(Japan)	1993	3	30	19.2	
Gokasho Bay					P.C.B.
(Japan)			20	106.9	(1994)
Uwajima Bay					P.C.B.
(Japan)			20	46.8	(1994)
Soerfjorden[1]	1991	10	30-60	100	Instanes
(Norway)					(1994)
Eagle Harbor	1993	17	100	99.1	Gilbert
(US)		13	100	117.4	(1994)

[1] Geotextile was installed
[2] Sand capping after dredging

Hiroshima/Kure Site

The chemical and biochemical aspects of this site are found in Kuroda and Fujita (1981), Fujita (1980), Ichikawa et al.(1981), and Horie (1991). The sand capping project was conducted in two phases. Phase 1 was conducted in 1979 covering an area of 160 m×120 m. Phase 2 was conducted a year later and covered an area 2.3 times that of Phase 1. Sand dredged from the nearby sea was used as capping material (mean diameter= 0.1-10 mm, G_s=2.62). The cap thickness was 50 cm and 30 cm, respectively, for phase 1 and 2. As shown in Figure C-2, the two sites overlap each other.

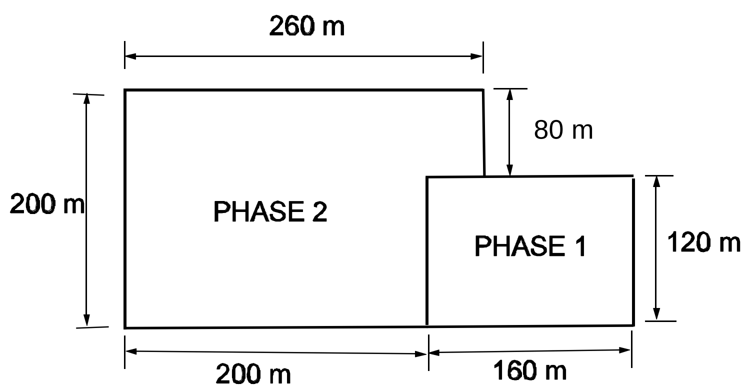

Figure C-2. Configuration of Hiroshima Site

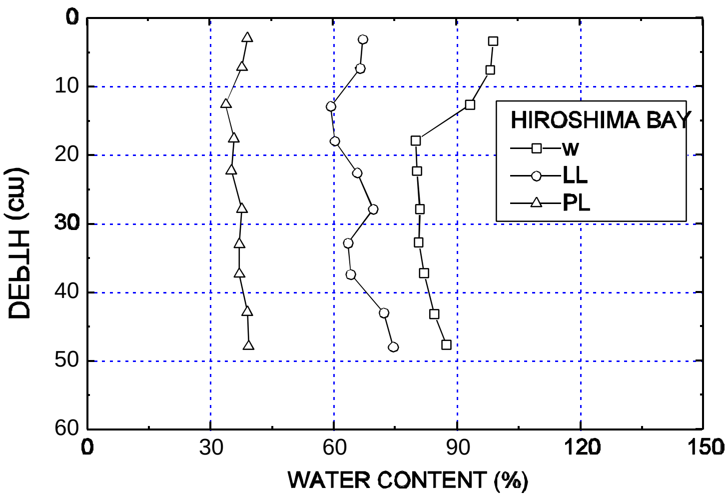

Figure C-3. Index Properties of Hiroshima Bay Sediments
(after Gomyoh et al., 1994)

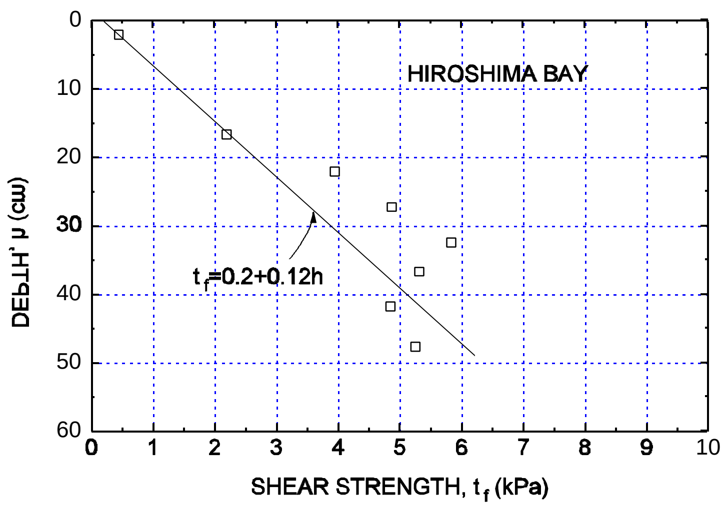

Figure C-4. Vane Shear Strength of Hiroshima Bay Sediments
(after Gomyoh et al., 1994)

The properties of the contaminated sediment were described by Gomyoh et al (1994). The natural water content of the first 10-20 cm of the mud is close to 100% (Figure C-3). The value at greater depths is about 80%, still higher than the liquid limit. Figure C-4 shows the typical value of vane shear strength distribution with depth. The undrained shear strength in the top 20 cm is extremely low, but increases linearly to 5 kPa as the depth increases to 50 cm. The sediments were slightly overconsolidated at the surface.

Matsushima Bay and Lake Biwa Sites

Toa Corporation conducted experimental projects at these two sites (Toa Corporation, 1994; Gomyoh et al., 1994). Sand capping at Lake Biwa covered an area of 110 m × 200 m. At Matsushima Bay, the project was composed of three areas, each 15 m×15 m. At Lake Biwa site, the upper 20 cm of sediment and then the area was covered with sand 20 cm thick. At Matsushima site, a 1.9 m thick sediment deposit was first dredged followed by a sand cap of 30 cm. The index properties of the bottom sediments at the sites are shown in Figures C-5 and C-6. Matsushima Bay mud has a natural water content as high as 250% The vane shear strength of these sites are given in Figures C-7 and C-8. The sediments at both sites show slight overconsolidated behavior. Piezocone penetration tests were conducted before and after dredging at Lake Biwa. It was reported that negligible strength reduction has resulted from dredging. Typical values of strength variation with depth are shown in Figure C-8. The sand used at Lake Biwa has a mean diameter of about 0.8 mm and a unit weight of 15.5 kN/m^3. Two types of sand were used at Matsushima Bay, one has a mean diameter of 0.25 mm and a unit weight of 11.7 kN/m^3 (dredged sand), and the other sand has a mean diameter of 0.45 mm and a unit weight of 15.6 kN/m^3.

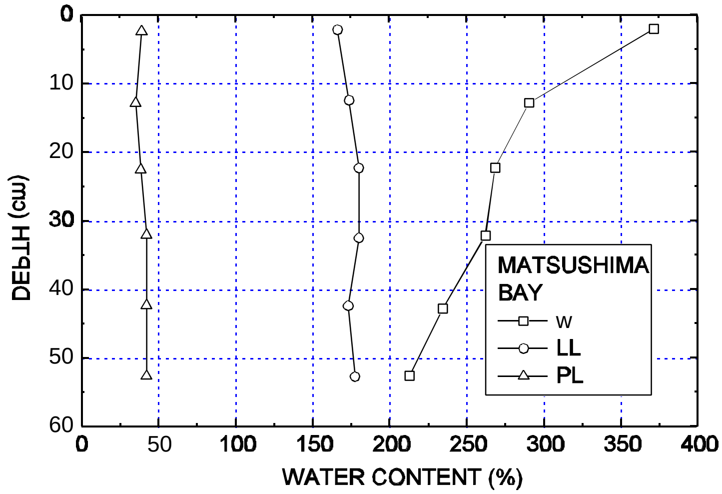

Figure C-5. Index Properties of Matsushima Bay Sediments
(after Gomyoh et al., 1994)

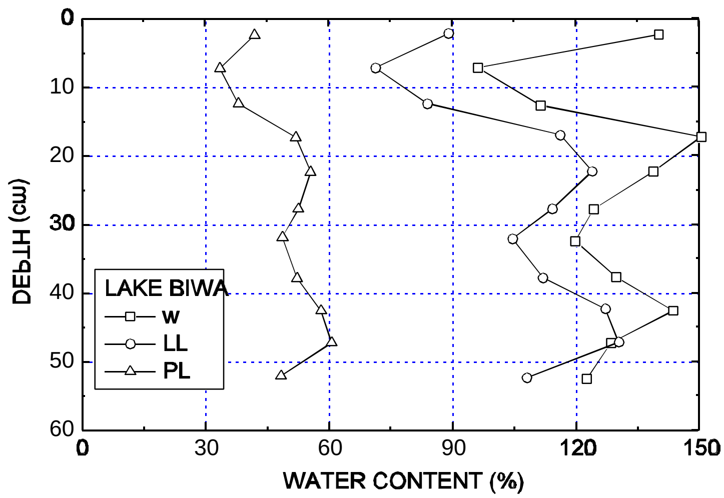

Figure C-6. Index Properties of Lake Biwa Sediments
(after Gomyoh et al., 1994)

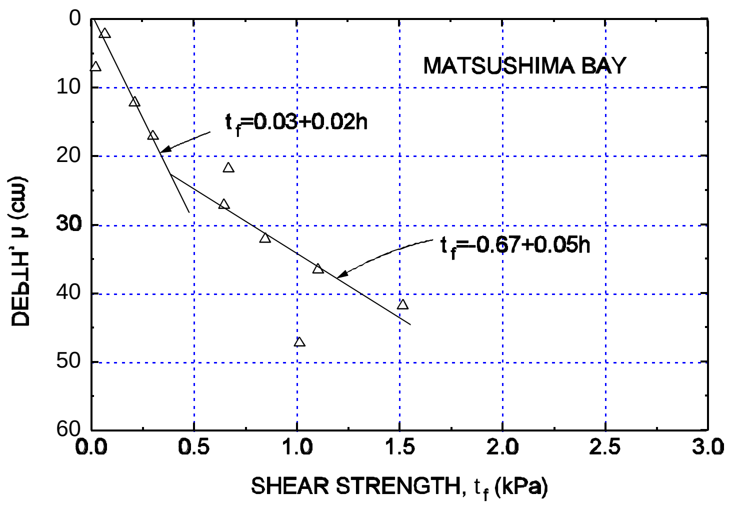

Figure C-7. Shear Strength of Matsushima Bay Sediments
(after Gomyoh, et al., 1994)

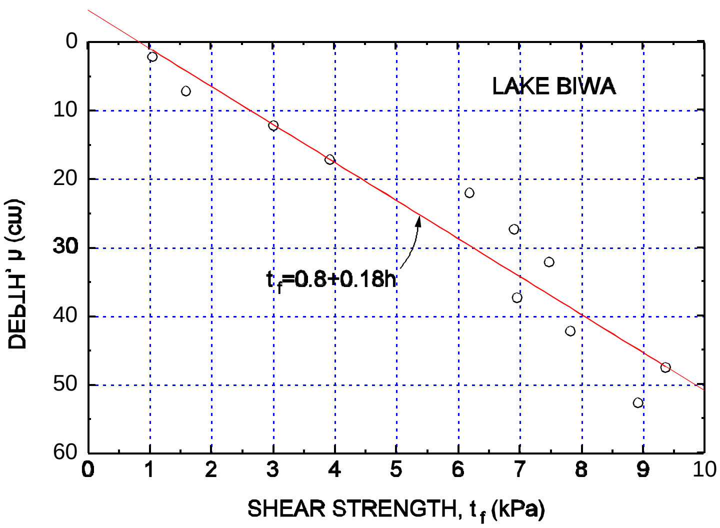

Figure C-8. Shear Strength of Lake Biwa Sediments
(after Gomyoh, et al., 1994)

Eagle Habor Site

The first in-situ sand capping project conducted in the US was that of the Eagle Harbor at the Wyckoff/Eagle Harbor Superfund Site (Figure C-9). The site was highly contaminated with mercury and polynuclear aromatic hydrocarbons. It was decided to cap two areas at the site with different materials. Areas 1 and 2 are at a water depth of 17 and 13 m, respectively. A split hull barge was used in Area 1 and the water jet washing of material off of a barge was used for Area 2. A sediment sample obtained at a point between Areas 1 and 2 shows that the sediments are comprised of 80% silt and 20% clay. Sediment properties were reported by Nelson, Vanerberden and Schuldt (1994) as LL= 40-50%, PL= 30%, G_s=2.65. The average unit weight of the sand cap was 16.4 kN/m^3. The targeted cap thickness was 1 m, but post construction surveying indicated slight variation of the final cap thickness over the site. Vane shear strengths obtained shortly after placement of the cap is shown in Figure C-10. These values are considerably higher than most Japanese sites. The measured in-situ shear strength indicated that the sediment is overconsolidated.

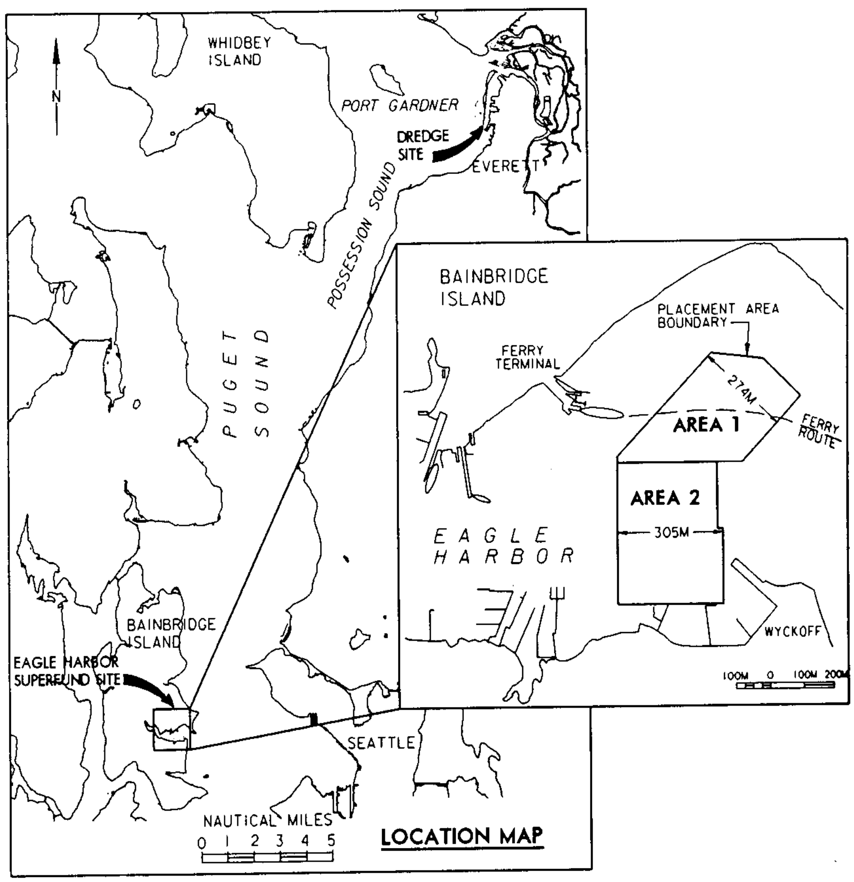

Figure C-9. Eagle Harbor Site (U.S.A.)

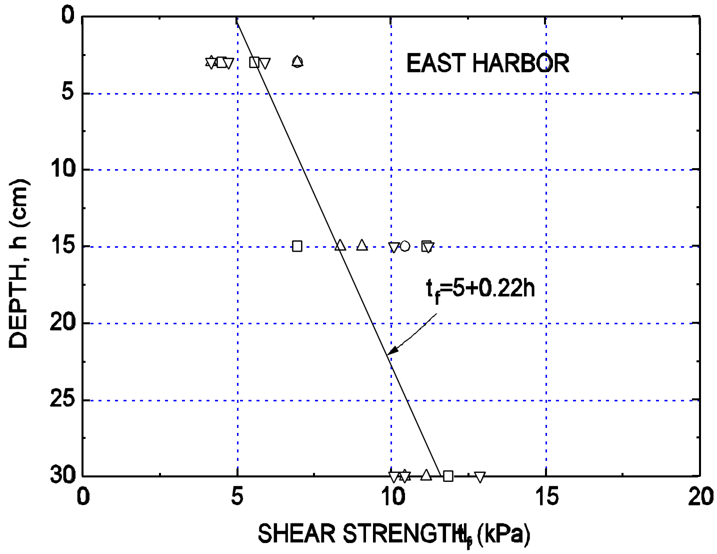

Figure C-10. Shear Strength of Eagle Harbor Site Sediments (after Gilbert, 1994)

In-situ Sand Capping Utilizing Geosynthetic: Case Histories

In-situ sand capping may not be feasible if the submarine sediment is extremely soft to the point where the sediment is not capable of supporting a cap. The geosynthetic sheet, placed between the cap and the soft sediment, allows the sand cap to be constructed over the soft foundation. With the geosynthetic in place, sediments may consolidate under the sand cap load and gain strength. The sand cap restrains the geosynthetic sheet and prevents migration of contaminated fines into the water column. Two successful projects, from Japan and Norway, are summarized below.

Minamata Site

Geosynthetics have been used in nearshore reclamation works in Japan since the 1960's (e.g., Fukuzumi and Nishibayashi, 1967; Watari and Higuchi, 1985). This experience led to a successful sand capping at Minamata site. The sediments at this site were highly contaminated with mercury. Human consumption of contaminated fish from this area led to the well known Minamata disease. It was decided that the sediments with mercury concentration greater than 25 ppm were to be dredged and capped. Hirose and Yamaguchi (1990) reported on the general aspects of this project.

A schematic drawing of Minamata site is shown in Figure C-11. It has an extremely soft sediment layer between 4.3 - 6.8 m deep. Some of the index properties are: G_s=2.71, LL=96%, PL=38.5%, PI=57.5% (Umehara and Zen, 1981). Figure C-12 shows the typical variation of strength with depth. The shear strength for this site is considerably lower than other Japanese capping sites. It exhibits normally consolidated behavior. Geotextile sheets, with a tensile strength of 78 kN/m and a hydraulic conductivity of 4.4×10^{-2} cm/s, were used. These geotextile sheets, each 30 m × 51 m, were laid over the dredged sediments with a 1 m overlay along the edges of the sheets to allow for possible differential settlement. Sand (ö=25°, ã=10 kN/m^3, D_{50}=0.1 mm) was spread in two layers under water. The water table was adjusted so that it was maintained at 50 cm during sand spreading. Water was then removed and the contaminated sediments were capped permanently with another type of sand (ã=14.7 kN/m^3, D_{50}=0.7 mm), 2 m thick, on top.

Soerfjorden Site

Geosynthetic was used in a sand capping project in Soerfjord, Norway (Instanes, 1994). The site was highly contaminated with heavy metals. The sediments have an undrained shear strength of 5-10 kPa and natural water content of 35%. The geosynthetic used was a composite material manufactured from polyester, density is higher than the water. It is comprised of a nonwoven geomembrane and a woven polyester geotextile which acted as separation/filter function and tensile reinforcement (Colins, 1994). The strength of geosynthetic was 50 kN/m. Polyester is denser than water, and thus, facilitated installation process. Fourteen geosynthetic sheets were placed with a minimum overlay of 2.5 m to allow for settlement. Finally, a sand cap of 30-60 cm was placed.

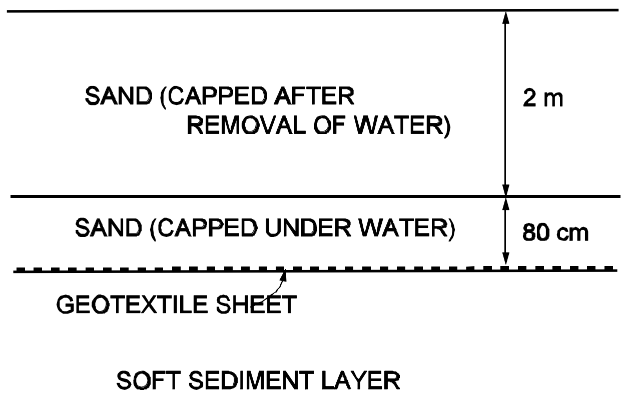

Figure C-11. Configuration of Sand Cap at Minamata Site
(after Namba, 1994)

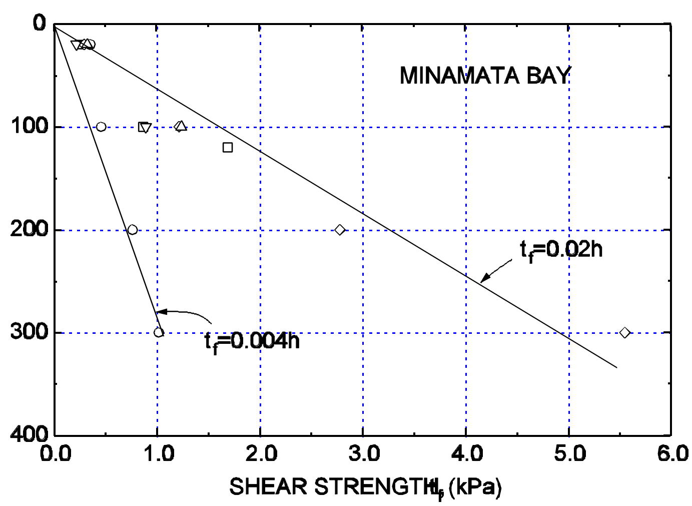

Figure C-12. Shear Strength of Minamata Site Sediments
(after Namba, 1994)

Geotechnical Considerations

In a capping project, there are several objectives to be considered regarding the cap thickness. For example, the sand cap should be sufficiently thick to offer chemical isolation, protection from intrusion as the result of bioturbidity, and protection from breach as the result of erosion. From a geotechnical view point, a larger cap thickness may lead to instability if the sediments have very low shear strength.

The cap stability and settlement due to consolidation are two main geotechnical issues. However, the most critical aspect of the cap would be its stability immediately after placement, before any excess pore water pressure due to the weight of the sand layer has dissipated. The settlement is related to long-term performance of the cap as the sediments consolidate simultaneously with the dissipation of excess pore water pressure while gaining additional strength. In this report, (*the (BF))* discussion will be focused on a short term stability analysis (i.e., the most critical state) of the sand cap as viewed from bearing capacity and slope stability analysis.

Bearing Capacity Analysis

In bearing capacity analysis, the sand cap is considered as a footing acting over large area. The footing contact pressure is replaced by an equivalent surcharge, q, due to the cap's effective unit weight, ã‘, and thickness, h. That is,

$$q = ã' h \qquad (1)$$

In undrained analysis, considering local shear failure (i.e., punching mode of failure) and a footing embedded on a purely cohesive soil with zero depth into the foundation, the ultimate bearing capacity, q_{ult}, is determined as (Terzaghi and Peck, 1967):

$$q_{ult} = 2/3\ c_u\ N_c \qquad (2a)$$

and

$$N_c = (2 + ð\ ) \qquad (2b)$$

where c_u is the undrained shear strength, and N_c is the bearing capacity factor. The usage of local failure is justified in sand capping projects because the bottom sediments are soft, and therefore, do not allow the classical bearing capacity type of failure to occur.

In design, the allowable surcharge is obtained by reducing the ultimate bearing capacity by a safety factor, typically of value 3. Thus, combining Eqs. (1) and (2), the allowable cap thickness, h_{allow}, is determined as

$$h = 1.14\ \ c_u\ /\ ã' \qquad (3)$$

Assuming a typical value of ã‘= 5 kN/m^3 and c_u= 1 to 2 kPa, the allowable cap thickness is between 20 and 50 cm. This range of value explained reasonably the success of most sand capping projects.

It should be pointed out that traditional bearing capacity analysis (Eq. 3) assumes a constant value of undrained shear strength. However, the review of case histories indicates that soft sediments are having undrained shear strength that increases with depth. Therefore, it is recommended to sing a small value of c_u) and only when limited shear strength data of the foundation are available.

Cap Stability Analysis

Cap slope stability is analyzed using a computer program with the procedure proposed by Leshchinsky (1987) and Leshchinsky and Smith (1989). It is based on a limit equilibrium approach considering a log-spiral and a circular failure mechanisms in the sand cap and soft sediments, respectively. If stability cannot be attained, the analysis will indicate whether geosynthetic reinforcement is needed or whether additional consolidation must be allowed to occur prior to cap placement. The analysis determines the geosynthetic strength required to restore stability if the safety factor falls below a specified value. The notation used is shown in Figure C-13.

Since the in-situ cap is completely submerged, the buoyant unit weight (ã‘) and the design value of the internal friction angle of the sand (ö$_d$) are specified. The water depth above the cap does not affect its effective stresses (and thus the cap stability) if external forces, such as waves, are not excessive. Different layers of sediments having depth (d_i) and undrained shear strength (c_m=c_u/F_s, F_s: safety factor) may be specified. The strength of each layer can be specified as a constant value or varying linearly with depth (Figure C-14).

It should be noted that the water depth affects the falling velocity of sand particles placed in water leading to different impact energy as they reach the sediments. This may result in different penetration depths into the soft sediments and affect the unit weight of sand. The shear strength of sand is also affected by this unit weight. However, since accurate identification of subaqueous material properties is very limited, it seems justified to ignore these effects at this stage. That is, the quantification of properties is not warranted considering the potential uncertainties in design.

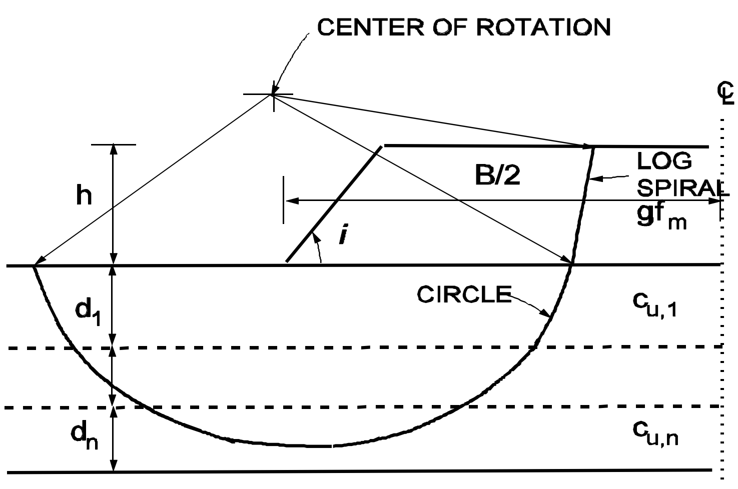

Figure C-13. Cap Stability Analysis

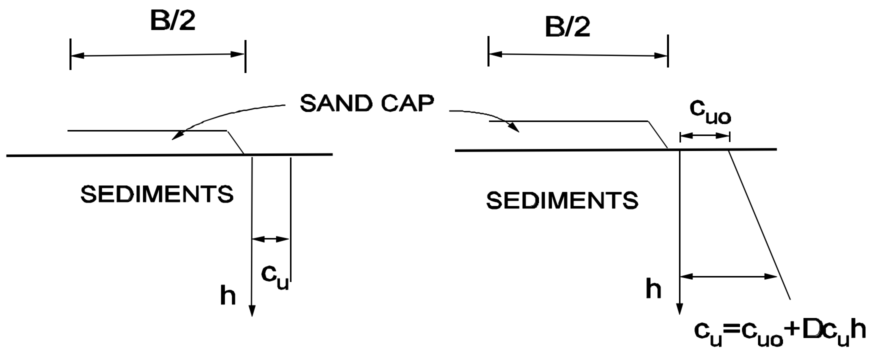

Figure C-14. Variation of Undrained Strength with Depth

Stability analyses were conducted for several of the reported case histories (Hiroshima, Minamata, Lake Biwa, Matsushima, and Eagle Harbor). Sediment properties required for stability analysis are available only at these sites (Table C-2). The internal friction angle of the sand and the slope angle of the cap are assumed as 35^o and 30^o, respectively. This is by assuming the largest possible angle of repose under water since the actual value was not available. Since the submerged unit weight of sand at the Hiroshima site is not available, it is assumed as 6.0 kN/m^3 in the analysis.

Table C-2 shows the sand cap thickness analyzed using a safety factor of 1.0 applied to the soils. Consequently, the calculated cap thickness signifies the maximum theoretical cap thickness. The analysis shows that Hiroshima site, Lake Biwa and Eagle Habor sites are stable against potential failure. In particular, the Eagle Harbor site has an extremely large safety margin. The analysis indicates that the Minamata site requires the sand cap to be placed with the aid of geosynthetic. The required geosynthetic strength is 7 N/m based on ã‘= 0.2 kN/m^3. If ã‘ is assumed as 6.0 kN/m^3 (i.e., a reasonable design value), the required geosynthetic strength increases significantly to 3.2 kN/m. The analysis also indicates possible instability of the sand cap at the Matsushima site. The required geosynthetic strengths are 4 N/m and 77 N/m for ã‘= 1.9 kN/m^3 and 5.8 kN/m^3, respectively. The successful sand cap placement at this site could have been due to dredging away of the top extremely soft sediment layer so that the actual sediments strength was larger than that used in the analysis. That is, dredging the top 20 cm of the sediment exposed the stronger sediment layer as foundation for the sand cap without the use of geosynthetic reinforcement.

Table C-2. Computed and Constructed Sand Cap Thickness

Capping Site	**Undrained Strength of Contaminated Sediments C_u = C_{uo} + Äc_u x depth**		**Effective Unit Weight of Cap Material**	**Constructed Cap Thickness**	**Computed Cap Thickness (max stable thickness)**
	c_{uo} (kPa)	Äc^u(kPa/m)	ã´(kN/m^3)	(cm)	(cm)
Hiroshima	0.2	12	6.0*	50	58
Minamata	0	0.4	0.2	80	**
Lake Biwa	0.8	18	5.7	20	295
Matsushima	0.03	2	1.9	30	22***
			5.8	30	5***
Eagle Harbor	5	22	6.6	100	>17m

*assumed value
**construction is infeasible without reinforcement. In actual construction, geosynthetic reinforcement was used.
***actual sediment strength was likely greater than that before dredging.

Conclusions and Recommendations

Successful case histories related to in-situ sand capping projects are reviewed and presented. This technique has been evaluated and proved feasible from a geotechnical view point. At the sites where the sediments are of extremely low strength, geosynthetics of adequate strength and permeability can improve the stability of the sand cap. Dredging away the top layer (10 - 20 cm) may also be a feasible solution. There are several topics that need to be further studied so that in-situ capping technique and its design procedure may be verified and refined:

1. It appears that construction technique is an important factor in the success of a sand capping projects. Sand dumped in lumps may penetrate the soft sediments and may cause resuspension of contaminants into the water column. Conversely, “raining” the sand in layers will allow gentle spreading and result in a stable sand cap. It is recommended that laboratory model tests be conducted and the performance monitored and quantified. This should lead to an optimized construction procedure which takes the geotechnical properties of the sediments into account.

2. It is suggested to develop an analytical technique which may be used to predict the density of sands pluviated in water. Experimental work should also be conducted to verify the theory. The effect of soil grain size, water depth and foundation compressibility should be considered as the parameters in the analytical and experimental studies.

3. It is recommended that the roles of a geosynthetic (reinforcement, separation, and filtration) in maintaining the cap integrity be considered in future research. This should also be studied and quantified using a well-controlled experimental work, including “control tests” which do not have a geosynthetic layer.

4. A reliable procedure to estimate the in-situ distribution of sediments strength is needed.

5. Potential external forces, in particular waves, need to be included in future studies.

6. Finally, a versatile procedure which considers the deformations, generation and dissipation of excess pore water pressure from the sediments should be developed.

Acknowledgment

The study as described herein was funded by the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss. Prof. T. Yamagami of the University of Tokushima, Prof. H. Ochiai of Kyushu University, Prof. T. Pradhan of Yokohama National University, Drs. T. Horie and T. Tsuchida of the Port and Harbour Research Institute, Mr. T. Namba of Kumamaoto Port Construction Office, and Dr. H. Hanzawa of Toa Habor Works Co. Ltd., have assisted the authors in one way or another with both published and unpublished information related to the Japanese case histories. Dr. T. Collins of the Huesker, Inc., offered details related to the Soerfjorden Site. However, the authors are responsible for the accuracy of results presented.

References

Collins, T.G. (1994). Personal communications, Huesker, Inc., 11107A South Commerce Blvd., P.O. Box 411529, Charlotte, NC 28241.

Fukuzumi, R. and Nishibayashi, K. (1967). “Surface improvement method of soft grounds by vinylon sheets.” Procedings, 5th Annual Convenction of JSCE, 121-124. (in Japanese)

Fujita, K. (1980). “Submarine purification project in Seto Inland Sea.” Hedoro, 19(3), 15-28. (in Japanese)

Gilbert, P. (1994). Unpublished trip report: Visit to Eagle Harbor, Seattle WA, 23 March.

Gomyoh, M., Hanzawa, H., and Pradhan, T. (1994). “ Investigation and measurements associated with environmental improvement of inner bay areas by covering polluted soft mud with a thin sand layer,” Proceedings, First International Congress on Environmental Geotechnics, Edmonton, Canada, 241-246.

Hirose, M. and Yamaguchi, A. (1990). “Mercury-contaminated sediment disposal work in Minamata Bay,” Proceedings, Japan Society of Civil Engineers, 421(VI-13), 233-242. (in Japanese)

Horie, T. (1991). “Numerical modeling for the prediction of sedimentary improvement by sand capping over a contaminated seabed,” Journal of Hydraulic Research, 29 (6), 829-850.

Instanes, D. (1994). “Pollution control of a Norwegian Fjord by use of geotextiles,” *Proceedings*, Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, 1053-1056.

Kuroda, H. and Fujita, K. (1981). “Submarine environmental improvements by dredging and sand capping.” Proceedings, 6th US/Japan Expert Meetings, Tokyo, 281-310. (in Japanese)

Leshchinsky, D. (1987). “Short-term stability of reinforced embankment over clayey foundation,” Soils and Foundations, 27(3), 43-57.
Leshchinsky, D. and Smith, D.S. (1989). “Deep-seated failure of a granular embankment over clay: stability analysis,” *Soils and Foundations*, 29(3), 105-114.

Namba, T. (1994). Personal communication, Kumamaoto Port Construction Office, Japan.

Nelson, Vanerberden and Schuldt. “Eagle Harbor Superfund Project,” Proceedings of Dredging 94, The Second International Conference and Exhibition on Dredging and Dredged Material Placement, American Society of Civil Engineers, Lake Buena Vista, FL. 1994 Dredging 94

O’Connor, J.M. and O’Connor, S.G. (1983). Evaluation of the 1980 Capping Operations at the Experimental Mud Dump Site, New York Bight Apex, Technical Report D-83-3, USACOE, WES, Vicksburg, Miss.

Palermo, M.P., Randall, R.E., and Fredette, T.J. (1993). Guidance for Subaqueous dredged Material Capping, 3rd Draft, USACE, WES, Vicksburg, Miss.

P.C.B. (1994). Unpublished Report, The Third District Port Construction Bureau, Ministry of Transport, Japan. (in Japanese)

SAIC (1984). Results of Monitoring Studies at Cap Sites #1, #2 and the FVP Site in Central Long Island Sound and a Classification Scheme for the Management of Capping Procedures, SAI-84/7506&C38.

Terzaghi, K. and Peck, R.B. (1967). *Soil Mechanics in Engineering Practice*," 2nd Edition, John Wiley and Sons, Inc., N.Y.

Toa Corporation (1994). Unpublished Report E-1. (in Japanese)

Umehara, Y. and Zen, K. (1981). "Consolidation and sedimentation characteristics of extremely soft hazardous mud." *Proceedings*, 6th US/Japan Expert Meetings, Tokyo, 143-166. (in Japanese)

Watari, Y. and Higuchi, Y. (1985). "Behavior and analysis of geotextiles on very soft ground at earth filling work." *Proceedings*, 30th Geotechnical Symposium, JSSMFE, 47-50. (in Japanese)